EVALUATION OF A MODEL FOR PREDICTING TRANSVERSE CRACKING OF FLEXIBLE PAVEMENT

MICHIGAN
RESEARCH LABORATORY SECTION

MICHIGAN DEPARTMENT OF
STATE HIGHWAYS AND TRANSPORTATION

EVALUATION OF A MODEL FOR PREDICTING TRANSVERSE CRACKING OF FLEXIBLE PAVEMENT

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TABLE OF CONTENTS

ABSTRACT

Hajek and Haas have proposed a practical method of determining the transverse cracking potential of bituminous mixes. A study was conducted in Michigan comparing actual transverse cracking performance with that predicted by the Hajek-Haas method. The results indicated that no functional relationship exists between actual and predicted transverse cracking of Michigan's flexible pavements. It was found that transverse cracking in Michigan could have been essentially eliminated had the mix stiffness criteria suggested by McLeod been used. However, use of McLeod's method would exclude from use many stiff bituminous mixes which have given essentially crack-free service.

A limited laboratory study was conducted in an attempt to determine why the Hajek-Haas method failed to work in Michigan, and perhaps find out why many very stiff bituminous mixes have performed in an essentially crack-free manner. Results indicate that the tensile strength, as well as stiffness modulus, both influence a bituminous mix's ability to resist cracking. Since stiffness modulus and tensile strength are not necessarily related, it should be necessary to include both of them in models for predicting transverse cracking; Hajek and Haas included only mix stiffness. The tensile strength of the aggregate has a significant effect on the tensile strength of the bituminous mixes. Therefore, mix tensile strength should not be reliably estimated from asphalt cement properties alone. The data also indicate that bituminous mix stiffness can be lowered to such an extent that tensile strength of the mix will no longer influence transverse cracking potential. It is thought that high tensile strength is required for stiff mixes to perform in an essentially crack-free manner.

It is concluded that the transverse cracking can be essentially eliminated in Michigan's flexible pavements if the mix stiffness guidelines of McLeod are followed. However, these guidelines will reject some stiffer mixes which can also display essentially crack-free performance. Procedures similar to those suggested by Burgess, et al, are suggested for identifying stiffer bituminous mixes that can perform essentially crack-free.

INTRODUCTION

Low temperature shrinkage cracking of asphalt pavements has become a serious and costly problem throughout many northern states. A typical example of this condition in Michigan is found on I 75 between Grayling and Gaylord where during winter months, riding characteristics, accelerated pavement deterioration, and increased maintenance costs can be attributed to thermal transverse pavement cracking (Fig. 1). Although usually not considered load related, thermal cracking does tend to reduce pavement serviceability and, therefore, should be an important consideration in asphalt pavement design. Although pavement age, material characteristics, soil moisture, and temperature conditions are all suspected of having influence on thermal cracking, the magnitude of their effects is not known. It has been observed in Michigan that, while some pavements show considerable thermal cracking, adjacent areas of the same pavement may be crack-free. If the variables causing the difference between the cracked and non-cracked areas could be isolated it should be possible to control

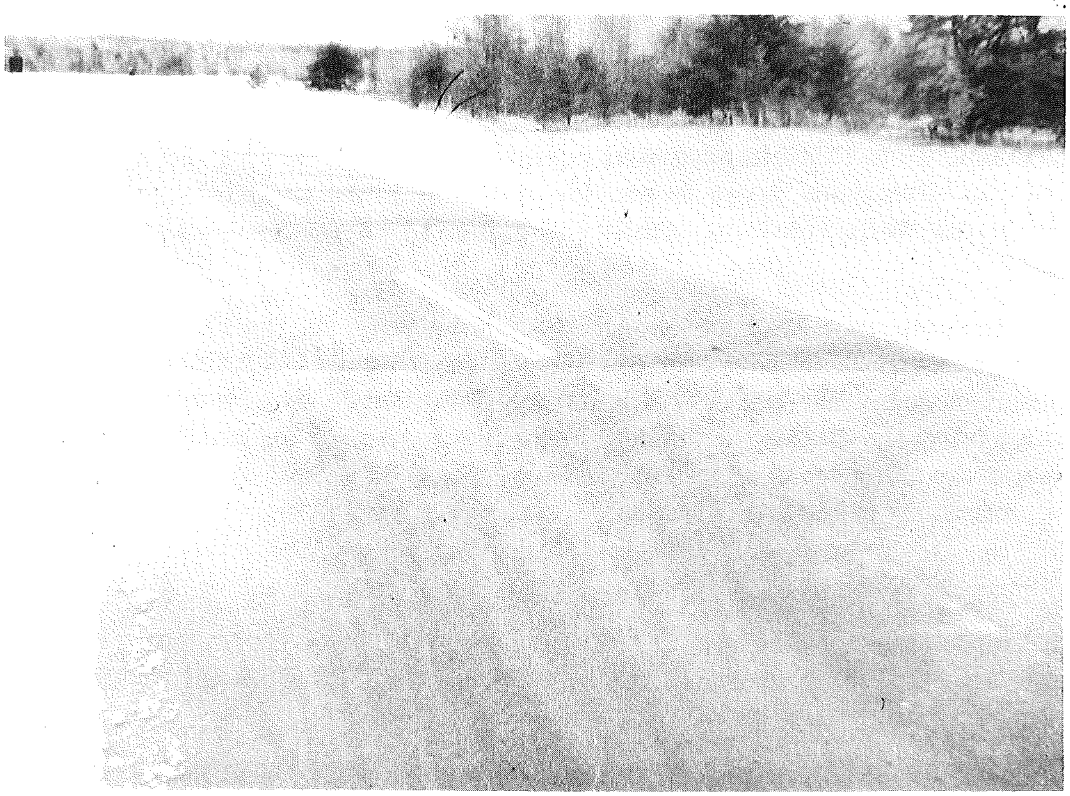


Figure 1. Typical example of transverse thermal cracking, I 75 between Grayling and Gaylord.

transverse cracking through proper design procedures. To do this, however, more information is needed concerning the materials properties which affect transverse cracking, the relationship between these properties and cracking frequency, and the reliability with which cracking frequency can be predicted.

Preliminary studies by the Research Laboratory indicated that considerable research had been conducted concerning the causes of thermal cracking in flexible pavements and several methods for predicting and controlling the cracking were suggested (1, 2, 3). Of these methods, that described by Hajek and Haas (3), appeared to be the more suitable for highway design purposes. This method places emphasis on prediction of the number of cracks to be expected for any design considerations. The design engineer, by using this method, would have the flexibility of designing a pavement to a permissible number of allowable cracks rather than to a "no-crack" condition.

The Hajek and Haas method is based on the use of a relatively simple mathematical model, or formula, which permits the prediction of the number of thermal cracks to be expected during a certain period of time, based on factors which are usually available from highway construction records. Of these factors it was indicated that stiffness of the asphalt would exert a major influence on thermal cracking frequency. Therefore, it was thought that this method might lead to design procedures whereby significant improvements in flexible pavement performance could be attained by designing the bituminous layer to provide a minimal low temperature cracking frequency while retaining stability at higher temperature.

In September 1973 a cooperative study between the Department of State Highways and Transportation and the Federal Highway Administration was initiated under the Highway Planning and Research Program to evaluate, more fully, the Hajek-Haas model as a suitable method for predicting transverse cracking of flexible pavement surfaces in Michigan. The specific objectives of this project as stated in the proposal were:

1) Determine if the Hajek-Haas model, which was developed for predicting thermal cracking of flexible pavement surfaces in Ontario, is suitable for similar use in Michigan, where construction procedures, climate, and materials might differ from those of Ontario.

2) Use data collected from this study to recompute constants of the model if this would make it more suitable for use in Michigan.

3) If the Hajek-Haas model is found to be capable of predicting Cracking Index, with reasonable accuracy, use it to develop a minimum allowable asphalt cement stiffness modulus for standard Michigan flexible pavement cross-sections.

4) Determine if the indirect method of estimating asphalt stiffness recommended by Hajek and Haas is reliable.

5) Should the model be found unsuitable, recommend the direction that any additional research should take if development of a usable model still appears warranted.

To accomplish these objectives a number of flexible pavements throughout Michigan were selected, for each of which the cracking index (number of transverse cracks per 500-ft segment of pavement) were to be estimated using the Hajek-Haas model. These estimates would be statistically compared with field measurements of the actual cracking condition of the areas. The suitability of this correlation would be used as the basis for directing further progress of the project.

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the FHWA. This report does not constitute a standard, specification, or regulation.

DESCRIPTION OF THE HAJEK-HAAS MODEL

The Hajek-Haas mathematical model (3) allows the prediction of the number of thermal cracks that may be expected in a bituminous surface during a given period of time based on the following factors, each of which are generally available from highway construction records:

1) Stiffness of the original asphalt used
2) Winter design temperatures
3) Thickness of the bituminous layer
4) Age of the pavement
5) Subgrade soil type.

Based on the testing and evaluation of more than 20 various functions, Hajek and Haas determined the functional relationship between cracking index and these other variables to be:

$$10^{I} = C_1 \times S^{(C_2 + C_3 t + C_4 a)} \times C_5^{d} \times C_6^{m} \times d^{C_7 s} \qquad (1)$$

where

I = the cracking index - number of full transverse cracks per 500-ft
s = stiffness of the asphalt, kg/sq cm
a = age of the pavement, years
t = thickness of asphalt, in.
d = type of subgrade, dimensionless code
m = winter design temperature, degree F
$C_1, \ldots, C_7$ are parameters of the model

The model equation was linearized to the form:

$$I = \text{Log } C_1 + C_2 \text{ Log } S + C_3 t \text{ Log } S + C_4 a \text{ Log } S + \text{Log } C_5 d + \text{Log } C_6 m + C_7 S \text{ Log } d \tag{2}$$

By means of the techniques of stepwise regression analysis (based on 32 data points) the parameters of this form of the equation were determined and the fitted equation, representing the mathematical model for prediction of low-temperature cracking frequency of asphalt concrete pavements, expressed as:

$$10^I = 2.497 \times 10^{30} \times S^{(6.7966 - 0.8740t + 1.3388a)} \times (7.054 \times 10^{-3})^d \times (3.193 \times 10^{-13})^m \times d^{0.6026\,S} \tag{3}$$

For the purpose of this model the cracking index is defined as the number of full and half transverse cracks per 500-ft section of two-lane highway.

APPLICATION OF THE MODEL TO MICHIGAN PAVEMENTS

In order to properly evaluate the model under Michigan conditions it was necessary to locate the study flexible pavement areas for which all of the five dependent variables are constant over a significant length of roadway, usually a section of pavement constructed under the same contract and uniform traffic volume. From the large number of pavements studied, 32 were finally selected as best meeting the desired requirements.

Most of the necessary information was obtained from construction records. The subgrade type was considered to be sand for all locations included in the survey because, as a general rule in Michigan, flexible pavements are built only on sand subgrades. Where short sections of fine grained subgrade may occur they are covered with a minimum of 25 in. of sand subbase.

The winter design temperature was determined on the basis of the freezing index as suggested by Hajek and Haas (3). For this purpose, the state was divided into four temperature zones developed from freezing index data (Fig. 2).

A survey of the selected field sites was made in which the number of transverse cracks per 500 ft of pavement section were determined using procedures described by Fromm and Phang (4). As pointed out by McLeod (5), crack counts will vary to some extent depending upon the person conducting the survey. However, this variation is not large and is proportional to the cracking frequency. Cracking survey data are considered to be quite reliable.

The estimated stiffness modulus of each test location was determined from penetration and viscosity data as outlined in Ref. (3). However, asphalts from different sources or batches were used so that differences in estimated stiffness modulus exist within a test area. A weighted mean stiffness modulus is reported in such cases. Construction records normally did not indicate the location within the pavement where each different source or batch of asphalt was used. Further, it was found that equal penetration viscosity data were not available for all projects.

The data collected from construction records and the field surveys are summarized in Table 1. Using Eq. (3) of the Hajek-Haas model the predicted cracking of these test areas were determined and the values compared with corresponding field measurements. Comparisons of test data obtained from the two methods are shown in Table 1 and Figure 3. The question at this point is whether or not the difference between the actual and the predicted cracking index (residuals) are of sufficiently small magnitude to accept the model as a reliable method of predicting cracking index.

If the model does predict the cracking index with accuracy, a plot of predicted values ($\hat{I}$) against measured values (I) should be a 45-degree straight line passing through the origin. From such a plot, shown in Figure 3, it is apparent that most of the $\hat{I}$ versus I values appear above the 45-degree ($\hat{I} = I$) line. Visually, it would appear that the difference between the predicted and actual I values is too large to justify acceptance of the model. To check this, however, the significance of the relationship was tested by statistical hypothesis.

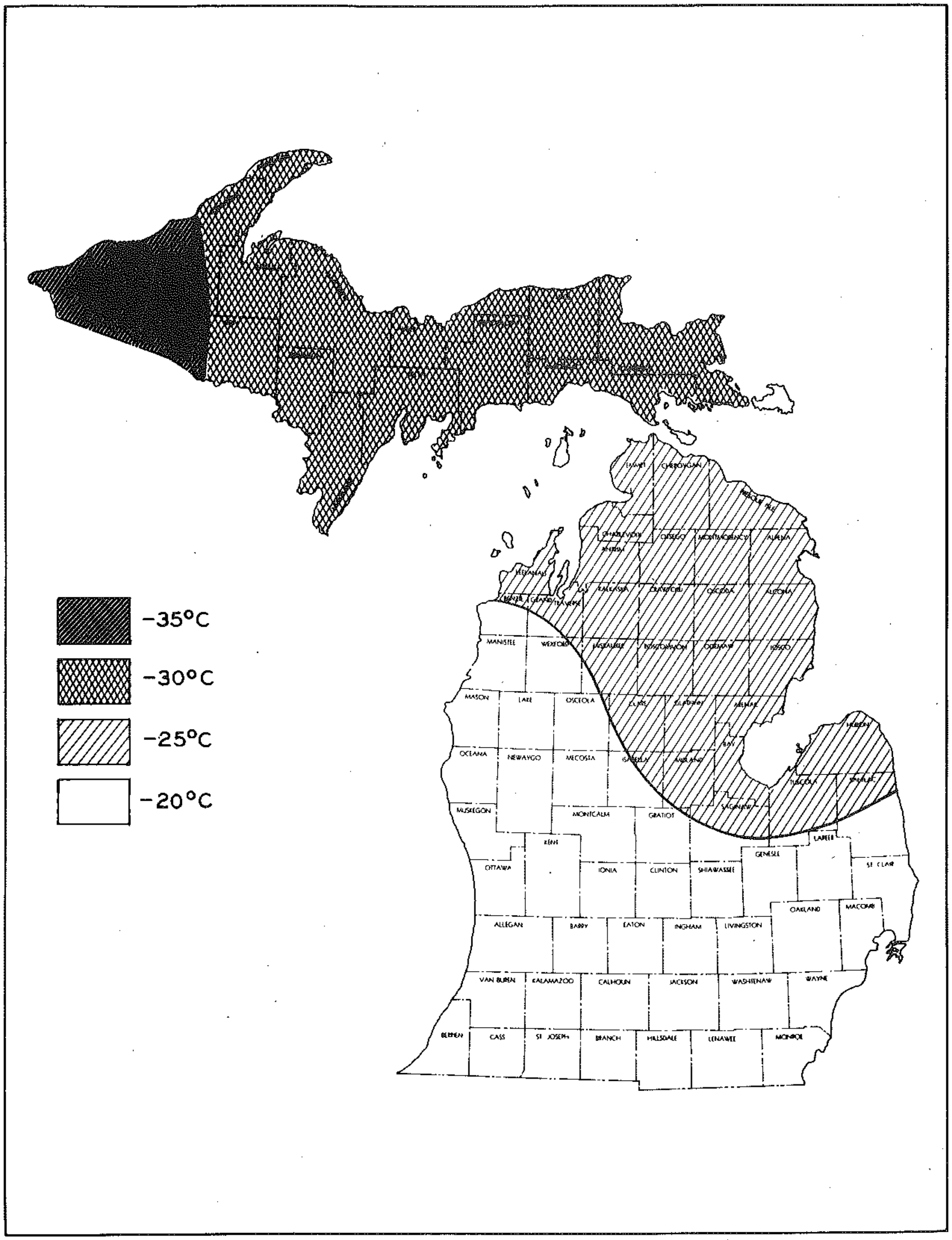


Figure 2. Temperature zones used to predict Cracking Index Values for Michigan, based on the Hajek and Haas definition of winter design temperature.

TABLE 1
SUMMARY OF CRACKING INDEX VALUES AND OTHER RELATED DATA

Observation Number	Cracking Index			Residual $\hat{I} - I$	No. of 500 ft Sub-Observations	Original Asphalt		Penetration Index	Base Temp, C	Winter Design Temp, C	Thickness of Pavement, in.	Age of Pavement, years	Estimated Stiffness of Asphalt Cement at 20,000 sec., kg/cm^2
	Mean Actual I	Standard Dev. of Mean Actual I	Mean Predicted $\hat{I}$			Penetration at 77 F	Viscosity at 275 F						
1	0.7	0.9	8.8	8.160	27	87	322	-0.64	46	-20	2.1	5	40
2	3.3	4.5	8.6	5.353	129	94	333	-0.39	45	-20	2.5	6	35
3	0.0	0.1	2.5	2.447	75	130	257	-0.38	43	-20	2.5	4	13
4	0.1	0.3	5.9	5.774	106	94	325	-0.44	46	-20	2.5	4	27
7	0.0	0.0	1.3	1.304	132	123	256	-0.36	38	-25	3.3	3	50
8	0.1	0.3	4.4	4.352	39	126	288	-0.26	44	-30	2.5	6	120
9	1.0	1.9	0.0	-1.007	76	228	180	-0.25	38	-30	2.5	4	35
10	0.3	0.7	0.0	-0.314	156	243	210	+0.09	38	-30	2.5	6	18
11	0.0	0.0	0.0	0.000	137	211	178	-0.40	38	-35	2.5	2	140
12	0.0	0.0	0.0	0.000	95	127	290	-0.25	42	-30	2.5	1	115
13	0.4	1.0	0.0	-0.368	95	223	222	-0.02	38	-35	2.5	6	100
14	3.4	4.2	2.7	-0.610	94	149	224	-0.56	41	-30	2.8	4	150
15	1.0	1.6	6.8	5.733	47	92	470	+0.07	48	-30	3.3	9	200
16	0.0	0.1	0.0	-0.027	75	231	186	-0.18	37	-25	2.5	5	60
17	0.0	0.3	12.0	11.920	63	91	344	-0.45	47	-25	2.5	8	100
18	0.0	0.0	4.7	4.741	64	231	191	-0.18	37	-25	2.5	5	60
19	0.0	0.0	0.0	-0.004	121	133	286	-0.09	43	-25	2.5	2	30
20	1.9	3.7	10.3	8.373	110	97	330	-0.40	46	-25	2.5	8	95
21	0.0	0.1	3.3	3.339	115	152	242	-0.32	42	-25	2.5	9	30
22	0.0	0.1	2.7	2.690	52	155	239	-0.27	41	-25	2.5	9	30
23	2.6	2.3	25.9	23.225	63	60	442	-0.59	51	-25	4.5	11	250
24	5.4	3.3	28.3	22.883	32	61	395	-0.73	51	-25	4.5	12	300
25	1.0	1.4	20.5	19.535	71	62	365	-0.84	49	-20	4.5	11	100
26	3.1	2.4	20.5	17.365	120	61	438	-0.60	50	-20	4.5	11	100
27	1.5	1.3	21.8	20.339	76	61	432	-0.61	50	-20	4.5	12	100
28	10.7	4.3	20.5	9.822	92	61	438	-0.60	50	-20	4.5	12	100
29	0.2	0.4	15.5	15.301	59	68	426	-0.53	50	-20	4.5	9	75
30	10.4	6.7	25.9	15.494	81	61	432	-0.61	50	-25	4.5	12	250
31	21.2	5.4	23.7	2.466	44	66	424	-0.56	50	-25	4.5	12	210
32	20.3	4.8	23.7	3.380	48	66	424	-0.56	50	-25	4.5	12	210
33	12.5	3.9	23.7	11.213	102	66	424	-0.56	50	-25	4.5	12	210
34	0.2	0.9	25.9	25.616	72	61	459	-0.53	50	-25	4.5	11	210

TABLE 2
FIELD SURVEY DATA FOR OBSERVATION 30 INDICATING
SUB-OBSERVATIONS USED FOR DETAILED STUDY

Station to Station	Actual Cracking Index		Estimated Asphalt Stiffness, kg/cm²	Predicted Cracking Index		Sub-Observations Cored
	1964	1973		1964	1973	
2018+00 to 2023+00	0	12-1/2				
2023+00 to 2028+00	0	11				
2028+00 to 2033+00	0	9				
2033+00 to 2038+00	2-1/2	20				
2038+00 to 2043+00	0	18-1/2				
2043+00 to 2048+00	0	20				X
2048+00 to 2053+00	0	8-1/2				
2053+00 to 2058+00	0	4				X
2058+00 to 2063+00	0	12				
2063+00 to 2068+00	0	4				
2068+00 to 2073+00	0	14-1/2				
2073+00 to 2078+00	1-1/2	9-1/2				
2078+00 to 2083+00	3	8-1/2				X
2083+00 to 2088+00	0	9				
2088+00 to 2093+00	1-1/2	22-1/2				
2093+00 to 2098+00	0	14				
2098+00 to 2103+00	0	22-1/2				X
2103+00 to 2108+00	0	21				
2108+00 to 2113+00	1	12-1/2				
2113+00 to 2118+00	1/2	16	300	11	29	
2118+00 to 2123+00	0	21				
2123+00 to 2128+00	0	10				X
2128+00 to 2133+00	0	9				
2133+00 to 2138+00	0	12				
2138+00 to 2143+00	1	7-1/2				
2143+00 to 2148+00	0	5-1/2				
2148+00 to 2153+00	0	10				
2153+00 to 2158+00	0	8				
2158+00 to 2163+00	0	8				
2163+00 to 2168+00	0	5				X
2168+00 to 2173+00	0	5				
2173+00 to 2178+00	0	11				
2178+00 to 2183+00	1/2	9-1/2				
2183+00 to 2188+00	0	15				
2188+00 to 2193+00	0	15-1/2				
2193+00 to 2198+00	0	24				
2198+00 to 2203+00	0	24-1/2				X
2203+00 to 2208+00	0	26				
2208+00 to 2213+00	1/2	19				
2213+00 to 2218+00	0	21				

TABLE 2 (Cont.)
FIELD SURVEY DATA FOR OBSERVATION 30 INDICATING
SUB-OBSERVATIONS USED FOR DETAILED STUDY

Station to Station	Actual Cracking Index		Estimated Asphalt Stiffness, kg/cm^2	Predicted Cracking Index		Sub-Observations Cored
	1964	1973		1964	1973	
2218+00 to 2223+00	0	24-1/2				
2223+00 to 2228+00	0	13-1/2				
2228+00 to 2233+00	0	15				
2233+00 to 2238+00	0	12				
2238+00 to 2243+00	0	6-1/2				X
2243+00 to 2248+00	0	14				
2248+00 to 2253+00	0	15				
2253+00 to 2258+00	0	12-1/2				
2258+00 to 2263+00	0	14-1/2				X
2263+00 to 2268+00	0	13				
2268+00 to 2273+00	0	4-1/2				
2273+00 to 2278+00	0	2-1/2				
2278+00 to 2283+00	0	1/2				
2283+00 to 2288+00	0	1				
2288+00 to 2293+00	0	1-1/2				
2293+00 to 2298+00	0	2				
2298+00 to 2303+00	0	2				
2303+00 to 2308+00	0	1/2				X
2308+00 to 2313+00	0	1/2				
2313+00 to 2318+00	0	3				
2318+00 to 2323+00	0	9-1/2	300	11	29	
2323+00 to 2328+00	0	8				
2328+00 to 2333+00	0	8				
2333+00 to 2338+00	0	8				
2338+00 to 2343+00	0	10-1/2				
2343+00 to 2348+00	0	8				X
2348+00 to 2353+00	0	0				X
2353+00 to 2358+00	0	6				
2358+00 to 2363+00	0	5-1/2				
2363+00 to 2368+00	0	1				X
2368+00 to 2373+00	0	5				
2373+00 to 2378+00	0	1				
2378+00 to 2383+00	0	1-1/2				
2383+00 to 2388+00	0	0				X
2388+00 to 2393+00	0	5				
2393+00 to 2398+00	0	7				
2398+00 to 2403+00	0	7-1/2				
2403+00 to 2408+00	0	15				
2408+00 to 2413+00	0	13				
2413+00 to 2418+00	1/2	15				
2418+00 to 2423+00	0	10				

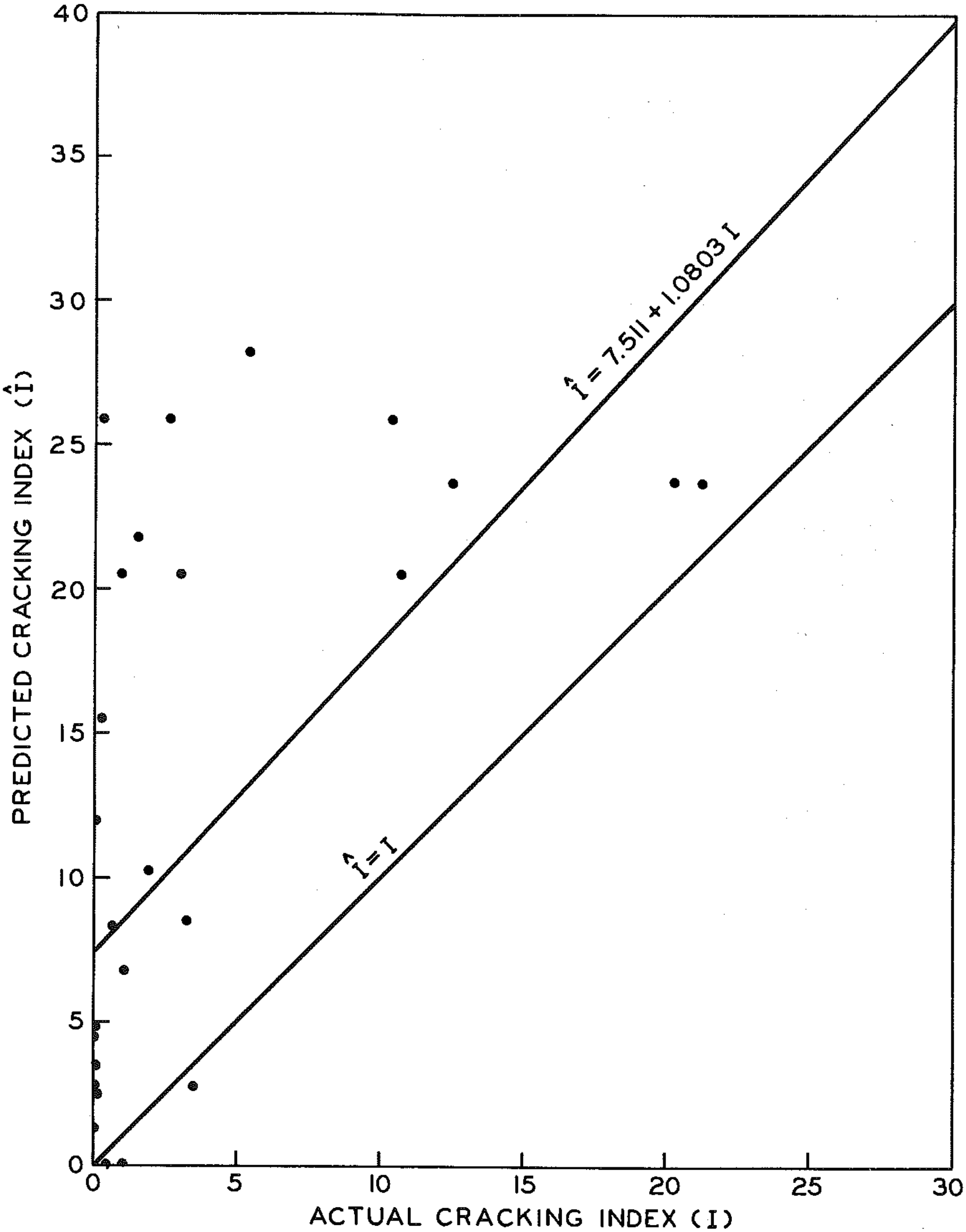


Figure 3. Relationship between predicted and actual cracking index values.

Using simple regression analysis to express (I) as a linear expression of ($\hat{I}$) we have

$$\hat{I} = 7.511 + 1.0803(I) \quad (4)$$

The correlation coefficient of I and $\hat{I}$ is 0.603 indicating that the linearity of I and $\hat{I}$ is not good. The statistical significance of this equation is more rigorously investigated by testing the null hypothesis:

H_0: Eq. (4) is the same as the equation $I = \hat{I}$.

against the alternative hypothesis:

H_1: Eq. (4) is not the same as the equation $I = \hat{I}$.

The test statistic F is computed to be 14.26068 which is much larger than $F_{0.05; 2, 3}$. So, the null hypothesis is rejected at the 0.05 level, and it is concluded that an Eq. (1) type model based on Michigan data would not be the same as Hajek-Haas' Eq. (3). From the plot in Figure 3, note that Eq. (4) is about parallel to the 45-degree line; therefore, the following null hypothesis was tested:

H_0: Eq. (4) is parallel to the 45-degree line.

against the alternative hypothesis:

H_1: Eq. (4) is not parallel to the 45-degree line.

The test statistic, t, is computed to be 1.56 which is less than $t_{0.05; 30}$. Therefore, we do not reject the null hypothesis and it is concluded that when Eq. (3) is applied to Michigan data it overestimates cracking index by 7.5 cracks on the average.

At this point it would be interesting to determine if the functional relation of cracking index to other variables, suggested by Hajek and Haas (3) could be used with Michigan data. According to the stepwise selection procedure, the fitted equation is:

$$I = 0.469 \text{ a} \log s - 0.856 \quad (5)$$

The multiple correlation coefficient is 0.6106 and the standard error of estimation is 4.45. The stepwise procedure indicates that every transformed variable except a log s contributes very little to the prediction of the cracking index. Again, the above fitted equation does not give a good prediction of the cracking index. That is, the functional relation given by Eq. (1) does not hold well for Michigan data.

Based on statistical analysis of the data collected in Michigan it is concluded that either the functional relationship among variables is not as proposed by the Hajek-Haas model, or that the Michigan data require a different formulation. Although the Hajek-Haas Eq. (3) apparently fits the Ontario data, due to the enormous flexibility afforded by the fitting parameters, when used with the Michigan data the fit deteriorates substantially.

In addition, the analysis indicates that the only significant factor of the model that affects transverse cracking is the stiffness of the asphalt cement acting in combination with aging (a log s in Eq. (2)). Therefore, little if any improvement in the reliability of the model is gained by including the other variables.

The statistical data show that the design engineer could expect considerable variation in mean cracking index of a pavement section from that predicted by the model. In addition to this variation there also should be variation in cracking frequency within the section as indicated by the large standard deviation of cracking index compared to its mean value. Because of the model's inability to accurately predict the mean frequency of occurrence of transverse cracking and because cracking frequency varies widely about the mean, the model will not reliably predict transverse cracking performance and its use in Michigan is not recommended.

LABORATORY STUDY OF ASPHALT PROPERTIES

The purpose of the laboratory testing portion of this project was to attempt to establish why the Hajek-Haas model was unreliable and to provide direction for possible future investigation.

Of the five independent variables used in the model, only asphalt stiffness could be correlated to transverse cracking to a significant degree. Unfortunately, however, the estimated stiffness value used to evaluate the model was the one variable whose accuracy is questionable. For this reason the poor reliability of the model could be due to unreliable estimated stiffness values rather than the model itself. It was felt necessary, therefore, to determine how well the estimated asphalt stiffness values used in the statistical model compared with actual stiffness values of the cracked pavement as determined by laboratory testing.

For this purpose, a cracked test area was selected for which construction records indicated a constant asphalt stiffness (300 kg/sq cm) for its entire length but for which the cracking index for each 500-ft length varied considerably -- a condition clearly indicating that either the estimated asphalt stiffness did not reflect actual in-place stiffness or that actual stiffness alone cannot be used to predict transverse cracking frequency.

Three 6-in. diameter cores of bituminous concrete were obtained from areas representative of high, medium, and low cracking index. Location and other information concerning the test samples are shown in Table 2. By confining the testing to a single area, all factors of the Hajek-Haas model, with the exception of stiffness, should be constant, thus permitting a direct comparison of stiffness and cracking index.

Testing Procedures

Part of the cored samples were used for extraction tests, which were performed by the Department's Testing Laboratory, and part were sawed into 1-1/4 by 1-1/4 by 4-in. blocks for stiffness testing (Fig. 4). This Figure shows the sample with metal end cover plates affixed for testing.

Initially it was planned to use the constant-rate-of-strain method for testing the stiffness of the samples, but several problems were encountered in attempting to use this method. The major problem was that a constant cross head speed did not yield a constant strain rate due, apparently, to increasing strain in the linkage connecting the sample to the upper and lower platens. At low temperatures particularly, linkage strain, which increases as stress increases, is very large compared to sample strain and varied from sample to sample so that calibration was impossible.

Another problem concerned the load frame which would not operate when placed in a freezer at the desired testing temperature of 0 ± 2 F. At this low temperature, the drive motor became overloaded and burned out after a few hours use. When a cooling chamber was placed around the sample only, serious frosting problems resulted. For these reasons it was necessary to change to the creep test method.

Considerable work using the creep test method to determine direct stiffness modulus has been reported by Burgess, et al, and this work was used as a general basis for our testing program (6). Figure 5 shows the equipment used and the sample in place for testing. Unfortunately, an insufficient number of cores were available to follow the test procedure described in Ref. (7).

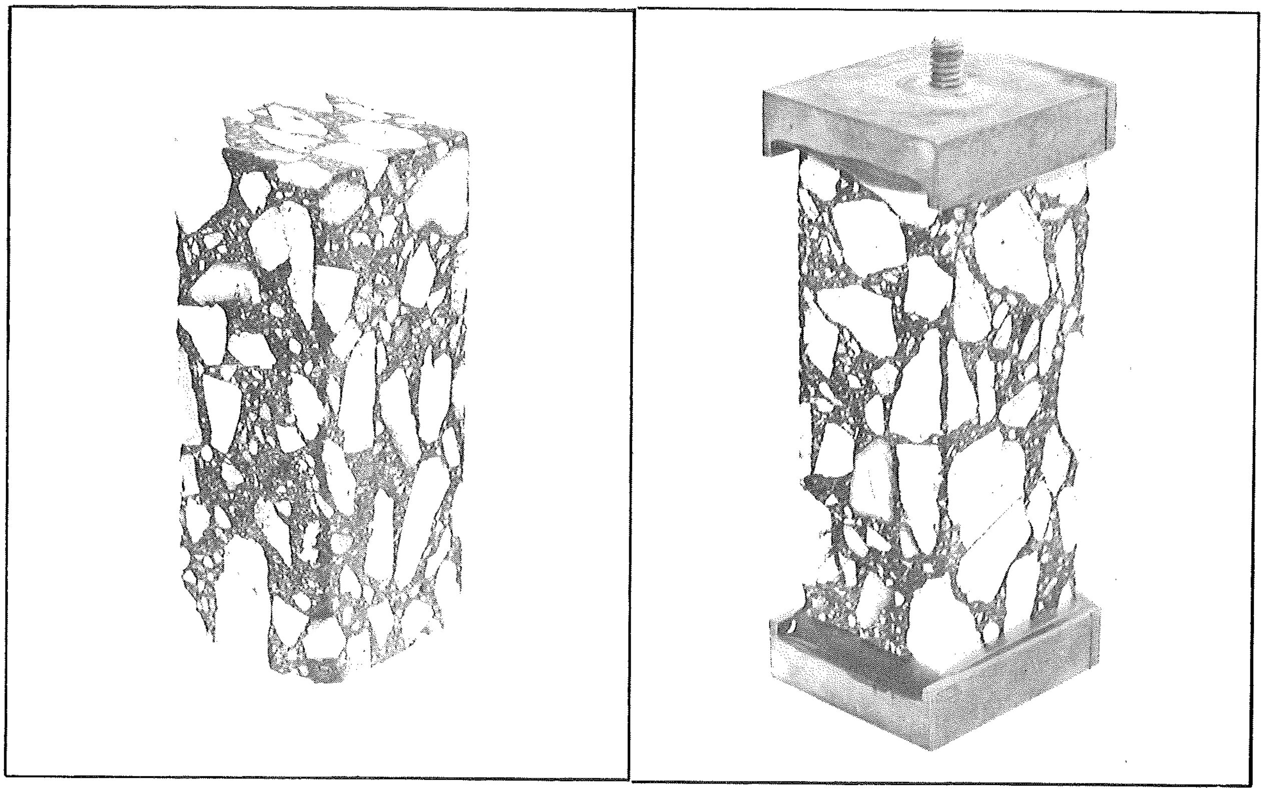


Figure 4. Typical 1-1/4 by 1-1/4 by 4-in. samples as sawed from 6-in. diameter cores and mounted in end caps for testing.

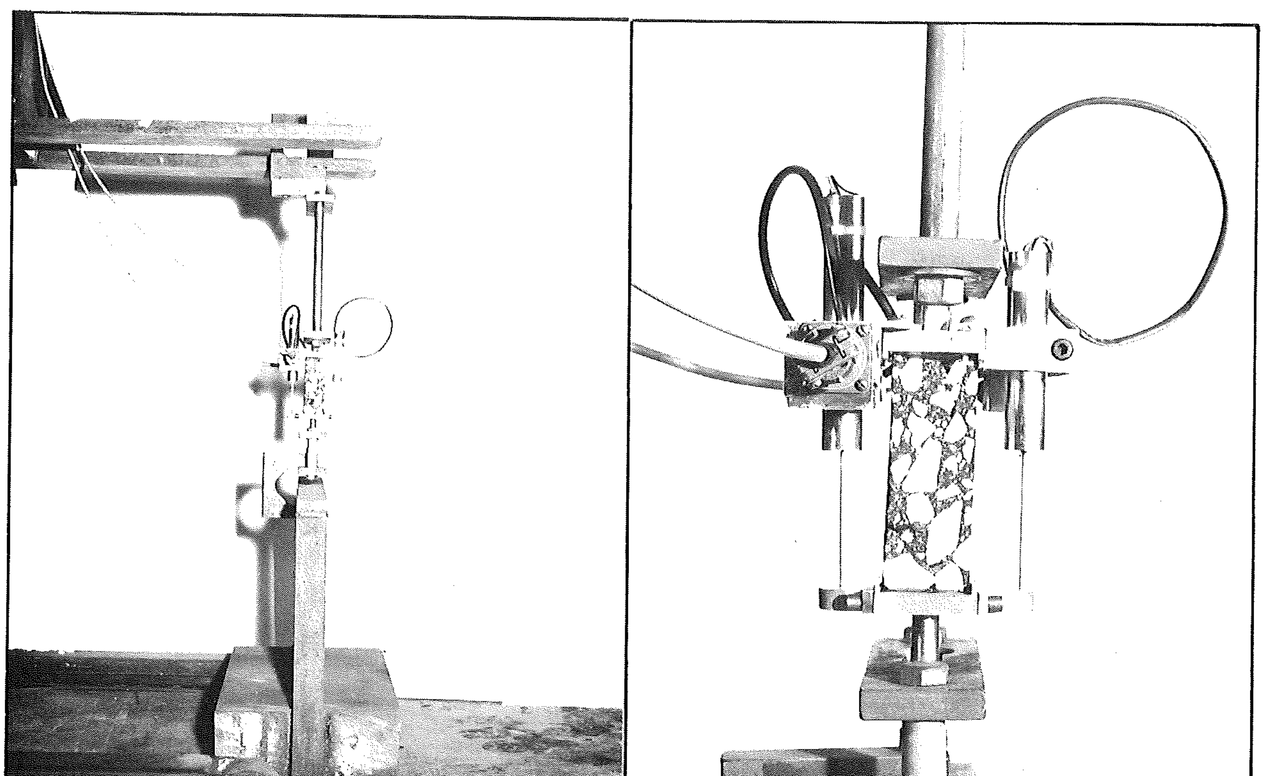


Figure 5. Test set-up for determining stiffness modulus.

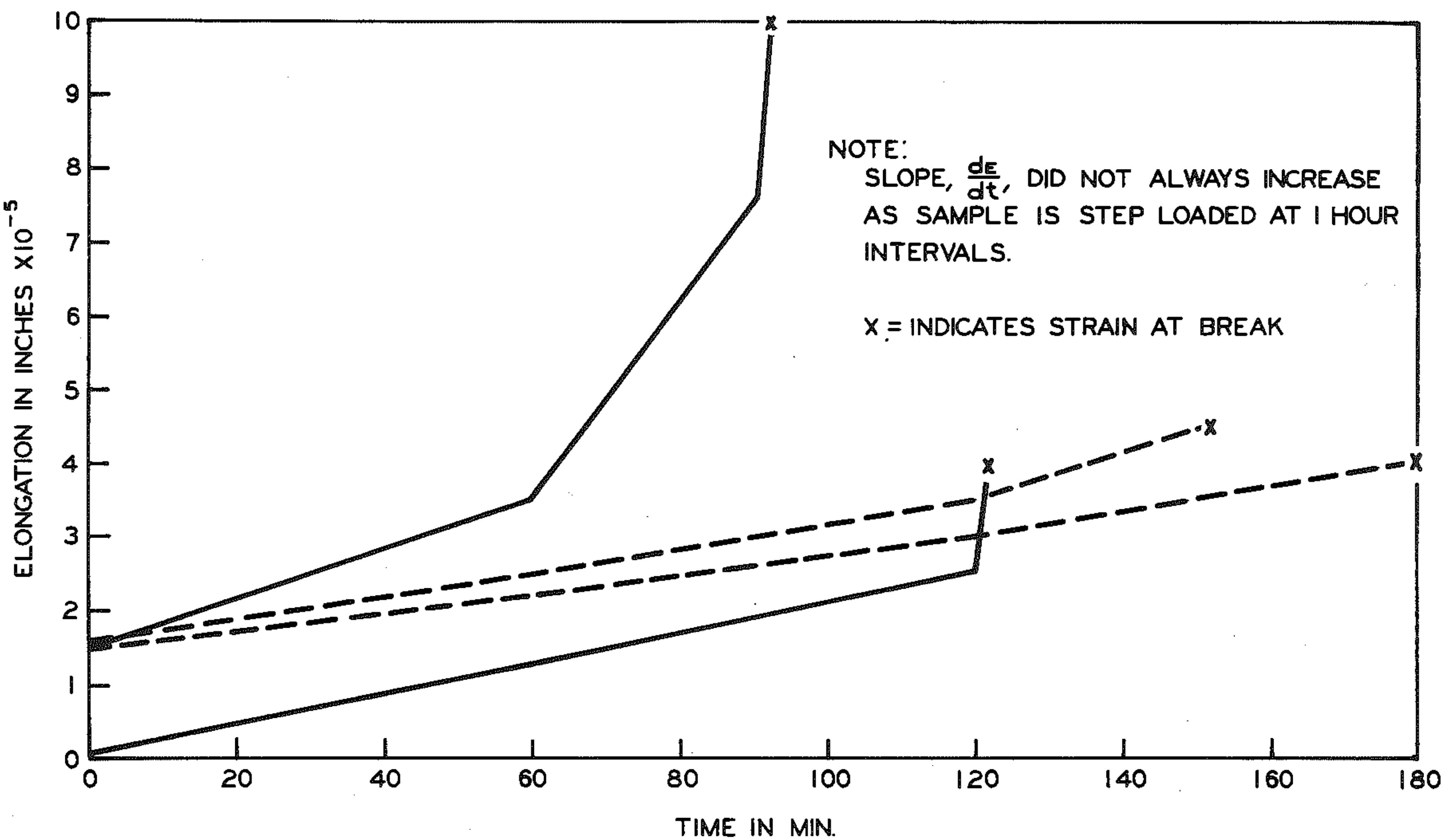


Figure 6. Typical time vs. elongation relationship.

TABLE 3
FIELD AGED ASPHALT CEMENT PROPERTIES - OBSERVATION 25-30

Observation Number	Core Number	Wearing Course							Leveling Course							Binder Course						
		Percent Bitumen	Penetration 25 C, dmm	Viscosity 275 F, cs	Ductility 25 C, cm	Air Voids, percent	V.M.A., percent	C_v	Percent Bitumen	Penetration 25 C, dmm	Viscosity 275 F, cs	Ductility 25 C, cm	Air Voids, percent	V.M.A., percent	C_v	Percent Bitumen	Penetration 25 C, dmm	Viscosity 275 F, cs	Ductility 25 C, cm	Air Voids, percent	V.M.A., percent	C_v
25	1, 2, 3	5.3	34	747	150+	0.9	13.0	0.88	4.7	21	910	28	5.8	16.7	0.88	4.8	38	570	150+	0.5	11.6	0.89
25	4, 5	5.1	39	560	150+	2.0	13.7	0.88	5.0	35	594	150+	4.3	16.0	0.88	5.1	52	449	150+	1.2	13.7	0.87
26	1, 2, 3	5.3	31	758	150+	0.6	12.9	0.88	5.0	24	873	116	4.4	16.4	0.87	5.0	33	668	150+	5.6	17.6	0.87
26	4, 5, 6	5.4	40	628	150+	0.9	14.6	0.86	5.2	26	813	150+	3.9	16.3	0.87	4.5	37	635	150+	2.1	13.1	0.89
27	1, 2, 3	5.4	42	635	150+	2.1	15.1	0.87	4.9	31	745	150	2.6	14.6	0.88	4.5	36	684	150-	1.0	12.1	0.89
28	1, 2, 3	6.1	19	1,170	10	3.0	17.7	0.85	5.1	16	2,030	8	9.7	21.1	0.87	4.8	19	845	10	3.1	14.1	0.89
29	1, 2, 3	5.6	18	1,100	7	2.5	15.5	0.87	5.0	16	1,100	8	8.6	19.9	0.88	4.8	21	772	117	0.9	12.9	0.88
29	4, 5, 6	5.3	44	571	150+	0.3	14.2	0.86	4.8	26	819	150+	4.6	15.8	0.88	5.0	32	710	150+	4.3	16.2	0.88
Mean, $\bar{x}$		5.4	33	771	119	1.5	14.6	0.87	5.0	24	986	95	5.5	17.1	0.88	4.8	34	604	128	2.9	13.9	0.88
Std. Deviation		0.3	10	236	62	1.0	1.6	0.01	0.2	7	446	68	2.4	2.2	0.01	0.2	10	244	49	1.9	2.0	0.01
30	1	5.2	31	808	150+				5.0	22	1,000	43				4.9	26	864	137			
30	2	5.0	31	784	126				4.9	27	848	88				4.6	33	750	150+			
30	3	5.2	26	886	108				4.4	19	1,050	10				4.4	24	943	73			
30	4	5.3	21	1,030	51				4.9	20	1,130	49				4.0	20	913	38			
30	5	5.4	21	1,070	58				4.4	19	1,280	7				4.7	29	802	150+			
30	6	5.2	30	824	125				4.4	26	916	150+				3.7	26	1,130	130			
30	7	5.4	26	888	95				4.6	25	920	82				4.7	40	603	150+			
30	8	5.3	32	777	150+				4.5	31	776	150+				4.8	35	727	150+			
30	9	9.1	17	1,280	9				4.5	24	1,060	65				4.9	34	764	150-			
30	10	5.1	20	802	61				4.5	22	1,010	113				4.9	28	884	150+			
30	11	5.2	29	884	150+				4.5	21	1,040	55				5.1	29	808	150+			
30	12	5.0	19	1,160	8				4.3	22	1,050	55				4.9	26	853	150-			
30	13	5.7	20	1,070	11				4.6	21	1,050	64				5.0	36	685	150-			
30	14	5.3	24	934	70				4.6	21	1,260	75				4.5	30	770	150+			
30	15	5.5	34	729	150+				4.8	28	823	150+				4.6	26	837	102			
Mean, $\bar{x}$		5.5	25	928	178				4.6	23	1,014	77				4.6	29	822	132			
Std. Deviation		1.0	5.5	160	370				0.2	3.5	144	46				0.4	5	123	34			

The test procedure adopted was to apply an initial stress to the sample such that the load could be sustained for at least a half hour. If the sample did not break during the first hour of loading the load was increased by approximately 100 lb and applied for another hour. This process was repeated until the sample finally broke.

The stiffness modulus was computed at one half hour, assuming that Maxwell's model applied. Attempts were made to calculate modulus values using the numerical solution for step loading suggested by Monismith (7) but these were not successful because strains were frequently so small, compared to the sensitivity of the LVDT instruments used, that instrument drift indicated negative strains at some stress levels. All stiffness testing was conducted at a temperature of 0 ± 2 F.

Tests were also made to determine air voids content and voids in mineral aggregate of the asphalt mixture in order that the volume concentration of aggregate (C_V) could be determined for typical Michigan bituminous mixes. A satisfactory design criteria for asphalt mixtures is considered to have a void content of about three percent and a C_V value of about 0.88. In these tests, the air void content for the wearing and binder courses were about 1.5 and 2.9 percent, respectively. The C_V value was approximately 0.88 for both layers so the mixes could be considered to be of good design.

Discussion of Test Results

Results of tests performed on the extracted asphalt cement are summarized in Table 3 and the creep test results in Table 4. Typical time vs. strain relationships are shown in Figure 6.

For each cored sample tested, penetration-viscosity data were used to estimate asphalt stiffness of each of the three bituminous concrete layers. These data are summarized in Table 5. The average stiffness modulus for the three layers and of the wearing course are related to the actual cracking index values as shown in Figures 7 and 8, respectively. The average temperature sensitivity (penetration index) of the three layers and of the upper layer or wearing course, are related to cracking index as shown in Figures 9 and 10, respectively. These figures clearly indicate that no functional relationship exists between the variables. Although limited, these data indicate that it is risky to assume that penetration viscosity data obtained from field-aged bituminous samples could be converted to asphalt cement stiffness which in turn could be used to estimate the transverse cracking frequency of a pavement.

TABLE 4
SUMMARY OF CREEP TEST RESULTS

	Core	Cracking Index I	Stress - PSI		Stiffness, PSI at 1/2 hr Loading Time $S_{1/2}$ hr*	Strain Rate at Failure	
			Initial σ_i	Breaking σ_{br}		Constant	Increasing
Binder Course	1	20.0	44	135	3.2×10^6		x
	1	20.0	45	94	---	x	
	2	4.0	44	136	9.8×10^6	x	
	2	4.0	46	184	12.0×10^6	x	
	5	5.0	45	128	0.4×10^6		x
	6	24.5	45	45	0.3×10^6	x	
	8	14.5	44	226	3.5×10^6		x
	8	14.5	44	135	---		x
	10	0.5	45	92	---		x
	10	0.5	45	138	---		x
	15	10.0	45	182	---	x	
	15	10.0	45	137	---		x
	$\bar{x}$	10.6	44.8	136.0	4.9×10^6		
	σ	8.1	0.6	47.2	4.9×10^6		
	4	22.5	88	175	4.0×10^6		x
	4	22.5	88	130	2.8×10^6		x
	5	5.0	91	91	---	x	
	6	24.5	92	172	0.4×10^6		x
	6	24.5	81	127	0.6×10^6	x	
	11	10.5	90	269	---		x
	12	0	90	127	1.8×10^6	x	
	12	0	89	132	2.4×10^6		x
	13	0	90	178	---		x
	13	0	89	162	0.7×10^6	x	
	$\bar{x}$	11.0	88.8	156.3	1.8×10^6		
	σ	11.3	3.0	48.4	1.3×10^6		
Wearing Course	1	20.0	133	356	14.4×10^6	x	
	1	20.0	138	368	13.4×10^6		
	2	4.0	136	363	19.4×10^6		
	2	4.0	135	360	8.9×10^6		
	6	24.5	136	274	11.2×10^6		x
	6	24.5	133	269	---		
	8	14.5	134	---	10.7×10^6		
	8	14.5	134	---	19.0×10^6		
	10	0.5	135	---	7.8×10^6		
	10	0.5	135	---	9.2×10^6		
	14	1.0	135	---	9.3×10^6		
	14	1.0	135	---	7.8×10^6		
	$\bar{x}$	10.8	134.9	331.7	11.9×10^6		
	σ	9.9	1.4	46.8	4.2×10^6		
	4	22.5	217	263	7.0×10^6	x	
	4	22.5	217	350	14.1×10^6	x	
	6	24.5	222	311	0.4×10^6	x	
	11	10.5	220	310	3.0×10^6	x	
	11	10.5	220	445	6.0×10^6	x	
	12	0	220	310	8.9×10^6		x
	12	0	219	353	3.6×10^6	x	
	$\bar{x}$	12.9	219.3	334.6	6.1×10^6		
	σ	10.5	1.8	57.2	4.5×10^6		

* Stiffness, $S_{1/2}$ hr, is not related to stiffness at breaking stress.

TABLE 5
PROPERTIES OF FIELD AGED ASPHALT CEMENT

Core Number	Penetration			Penetration Index			Base Temperature C			Design Temperature	Stiffness, kg/sq cm				Actual Crack Index
	Binder Course	Leveling Course	Wearing Course	Binder Course	Leveling Course	Wearing Course	Binder Course	Leveling Course	Wearing Course		Binder Course	Leveling Course	Wearing Course	Three Layer Mean	
25 - 1, 2, 3	38	21	34	-0.66	-0.59	-0.44	54	61	55	-20	300	900	250	483	4.0
25 - 1055, 1115	52	35	39	-0.68	-0.70	-0.69	51	55	53	↑	160	380	300	280	0
26 - 1, 2, 3	33	24	31	-0.61	-0.50	-0.52	56	60	58		275	600	475	450	8.5
26 - 4, 5, 6	37	26	40	-0.55	-0.54	-0.51	56	59	55		350	510	275	378	0
27 - 1, 2, 3	36	31	42	-0.50	-0.54	-0.47	56	58	55		300	480	225	335	0
28 - 1, 2, 3	19	16	19	-0.85	+0.02	-0.46	60	67	52		1,200	400	110	570	5.5
28 - 4, 5, 6	21	16	18	-0.79	-0.70	-0.56	59	62	61	↓	950	1,050	850	950	16.0
29 - 1, 2, 3	32	26	44	-0.57	-0.53	-0.57	57	59	54	-20	400	510	230	380	0
										$\bar{X}$ =	492	604	339	478	4.25
										σ =	372	242	230	211	5.73
										$\sigma/\bar{X}$ =	0.76	0.40	0.68	0.44	1.35
30 - 1	26	22	31	-0.46	-0.43	-0.44	59	61	58	-25	1,000	1,600	850	1,150	20.0
30 - 2	33	27	31	-0.48	-0.45	-0.48	57	59	58	↑	750	1,000	900	883	4.0
30 - 3	24	19	26	-0.40	-0.59	-0.43	63	61	60		1,800	1,750	1,300	1,617	8.5
30 - 4	20	20	21	-0.72	-0.44	-0.42	60	62	61		1,800	1,750	1,500	1,683	22.5
30 - 5	29	19	21	-0.51	-0.35	-0.38	58	63	62		1,000	1,600	1,600	1,400	5.0
30 - 6	26	26	31	-0.11	-0.38	-0.41	61	60	57		850	1,000	750	867	24.5
30 - 7	40	25	26	-0.92	-0.35	-0.43	53	60	60		3,000	1,100	1,250	1,783	6.5
30 - 8	35	31	32	-0.43	-0.49	-0.46	57	58	58		800	1,000	950	917	14.5
30 - 9	34	24	17	-0.41	-0.25	-0.41	57	61	64		800	1,050	2,000	1,283	0.5
30 - 10	28	22	20	-0.40	-0.41	-0.88	59	61	59		1,000	1,600	2,050	1,550	0.5
30 - 11	29	21	29	-0.50	-0.42	-0.37	58	61	59		1,000	1,600	950	1,183	10.5
30 - 12	26	22	19	-0.47	-0.37	-0.47	59	61	62		1,050	1,400	1,700	1,383	0
30 - 13	36	21	20	-0.49	-0.40	-0.51	56	62	61		700	1,700	1,650	1,350	0
30 - 14	30	21	24	-0.54	-0.17	-0.41	58	63	60	↓	1,050	1,350	1,250	1,217	1.0
30 - 15	26	28	34	-0.50	-0.49	-0.48	59	59	57	-25	1,100	1,050	800	983	10.0
										$\bar{X}$ =	1,180	1,370	1,300	1,283	8.53
										σ =	601	305	433	294	8.42
										$\sigma/\bar{X}$ =	0.51	0.22	0.33	0.23	0.99

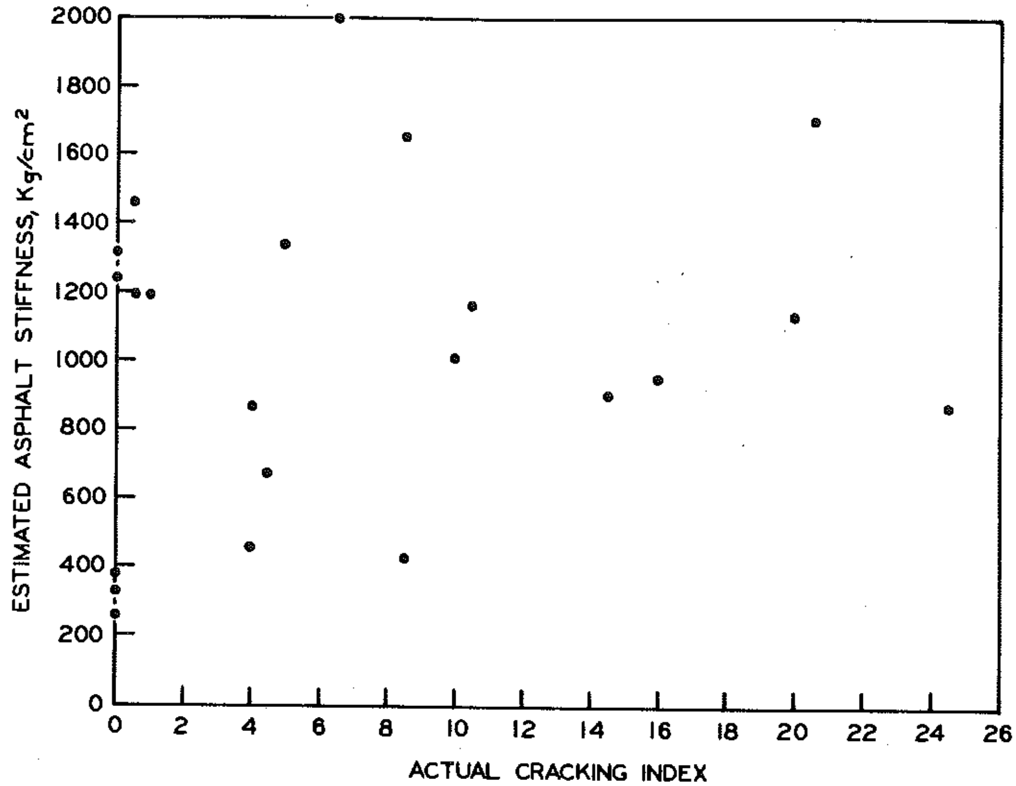


Figure 7. Average estimated asphalt cement stiffness of field-aged cores vs. actual cracking index.

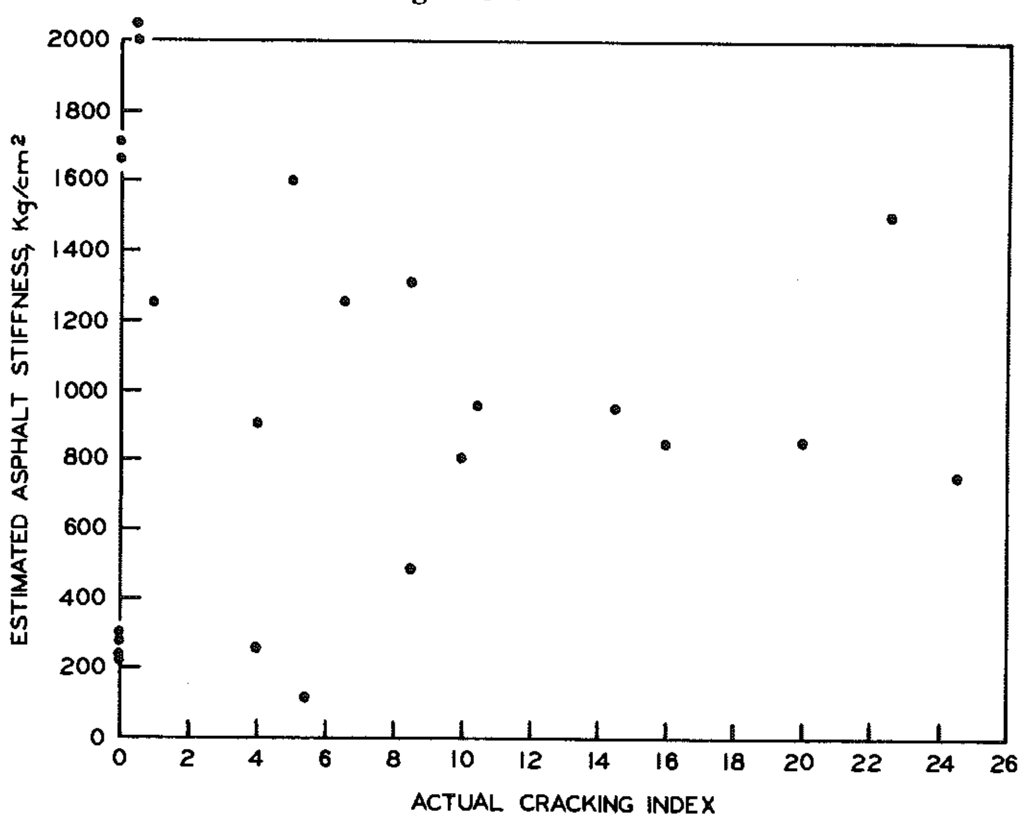


Figure 8. Estimated asphalt cement stiffness of the wearing course portion of field-aged cores vs. actual cracking index.

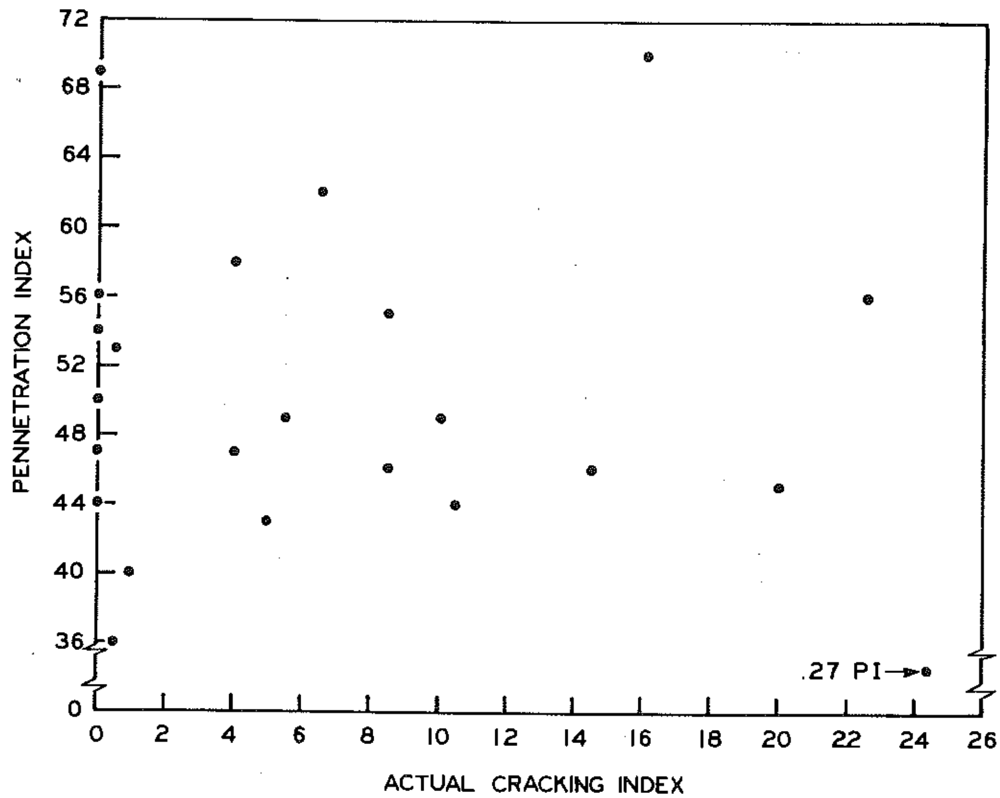


Figure 9. Average penetration index of entire field-aged cores vs. actual cracking index.

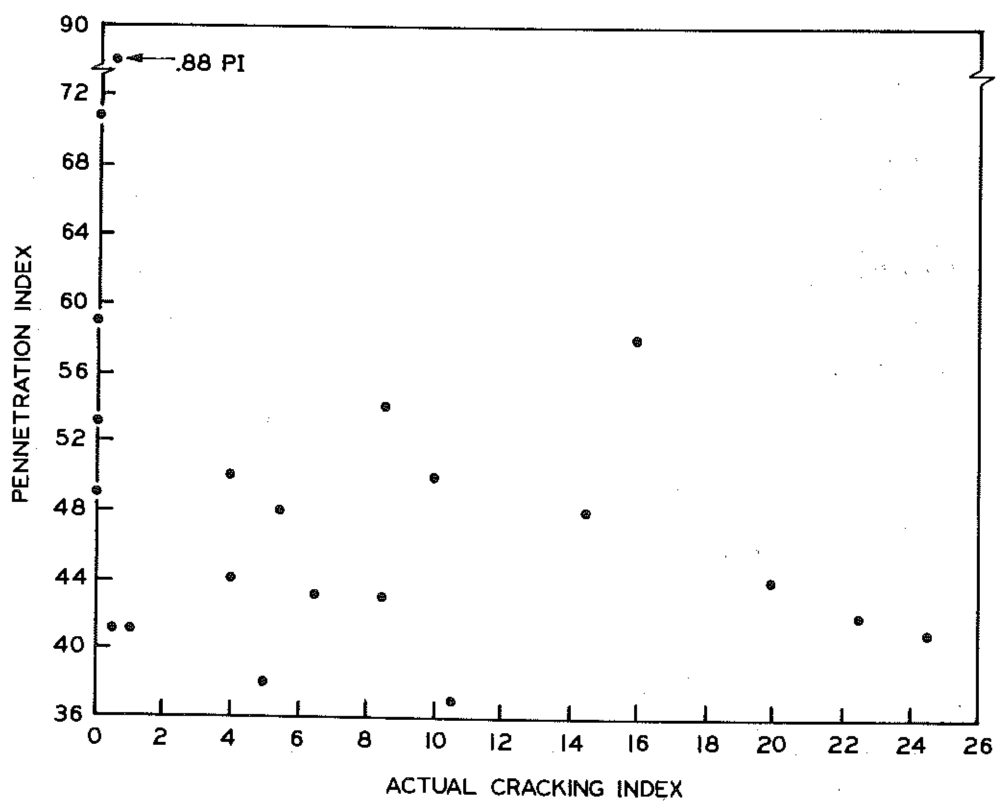


Figure 10. Penetration index of the wearing course portion of field-aged cores vs. actual cracking index.

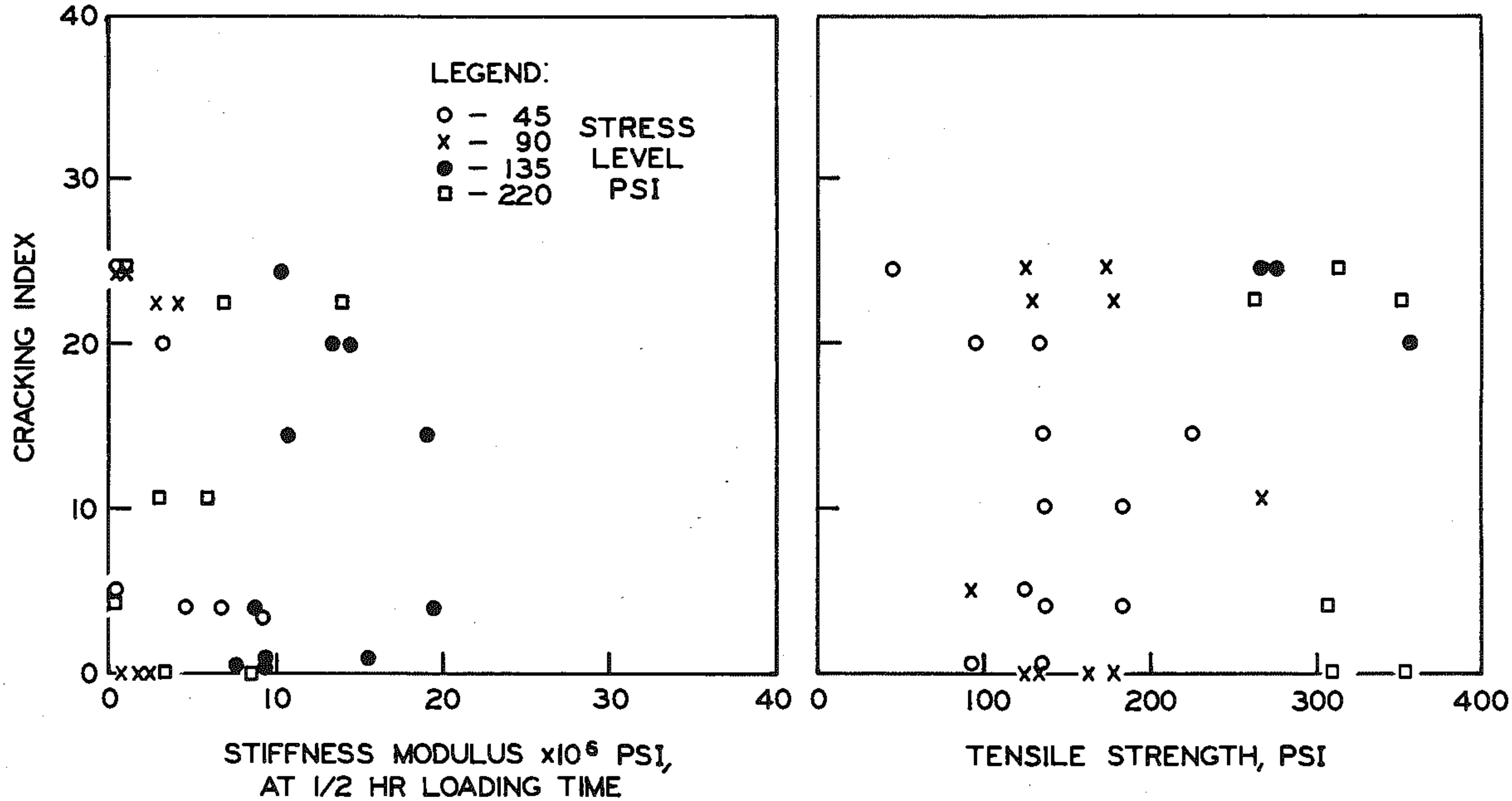


Figure 11. Relationship between cracking index and strength properties of wearing and binder course materials.

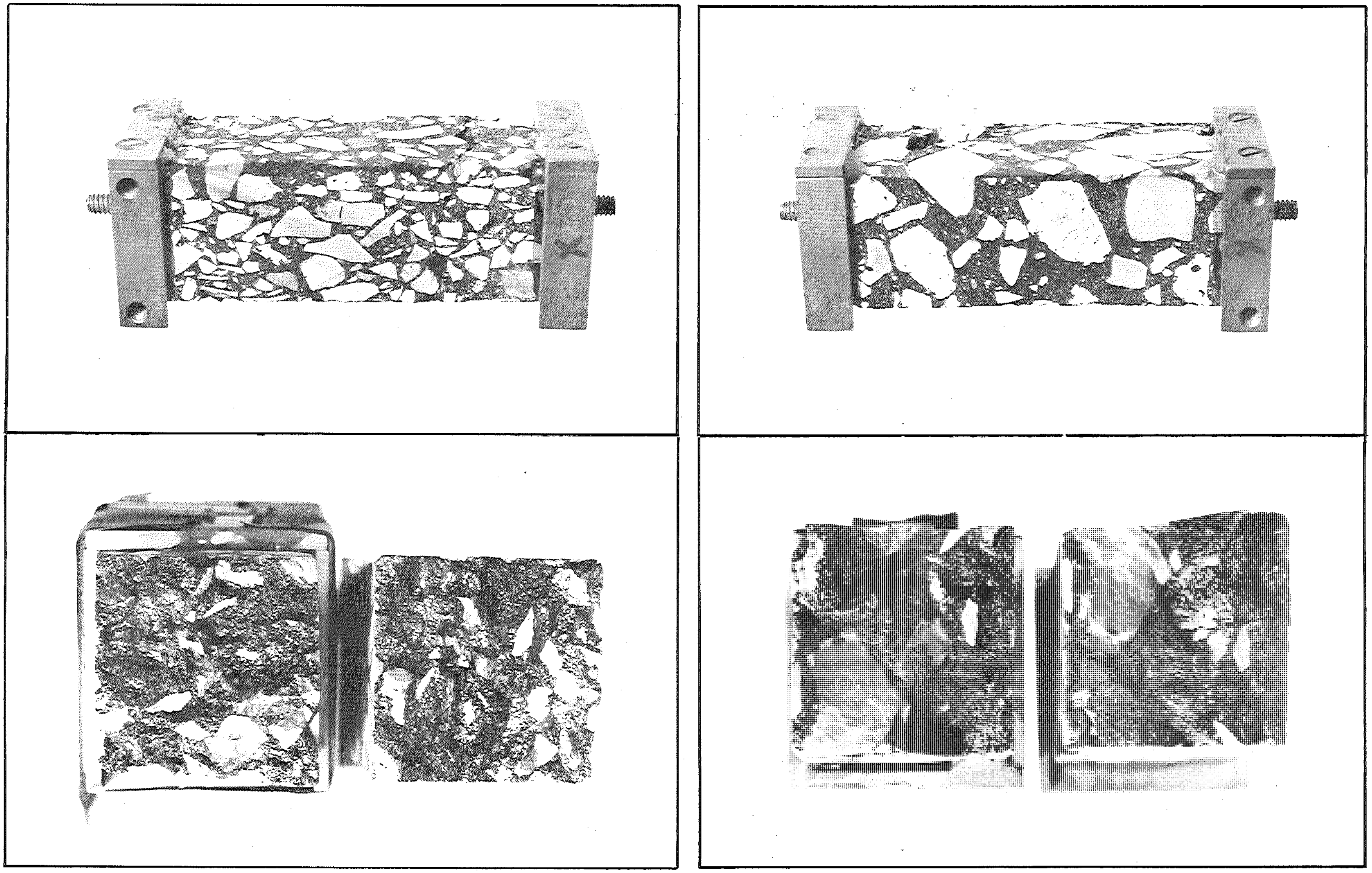
For observation 30, Table 5 indicates that the asphalt cement stiffness increased with age as was expected but it was surprising to see that the asphalt sensitivity, as indicated by the penetration index, decreased with age. In looking for relationships between asphalt cement stiffness and cracking index, Table 5 shows that the coefficient of variation of the cracking index is much larger than it is for asphalt cement stiffness. This is another indication that other factors in addition to asphalt cement stiffness strongly influence cracking index.

Figure 11 compares the half-hour mix stiffness and tensile strength with the measured cracking index. These data show that there is no usable correlation indicating that the cracking index could not be predicted on the basis of the data obtained from the creep test procedure used. The data do show that the tensile strength of the wearing course layer of the asphalt is, on the average, more than twice that of the binder course (Table 4). The difference is thought to be due to the mix design characteristics of the different asphalt layers, the wearing course having a higher asphalt content and a finer aggregate gradation than does the binder course. A bituminous concrete pavement layer, therefore, has greater tensile strength at the top of the layer. This variation in tensile strength throughout the depth of asphalt complicates predicting transverse cracking performance.

Any test used for predicting pavement cracking potential should include an accurate estimate of the tensile strength of the mix in addition to an estimate of mix stiffness since well designed mixes may not necessarily have the same tensile strength.

Heukelom (8) has shown the relationship between tensile strength and stiffness modulus of asphalt cement and indicates that the same relationship holds for bituminous mixes. The only difference between various asphalt cements and asphalt mixes is the magnitude of the tensile strength at any given asphalt cement stiffness at which a wide range of tensile strengths are possible depending upon the mix design. Those factors affecting the tensile strength of a mix are gradation of aggregate used, density, tensile strength of the aggregate, asphalt cement content, temperature, and possibly other factors. Figure 12 illustrates the importance of the strength of the aggregate itself in the tensile strength of bituminous mixes at low temperatures where the asphalt is stiff. For the sample shown, a large part of the fracture is through the aggregate particles.

Two distinctly different strain failure patterns were indicated from the creep tests. Some of the samples broke suddenly, with no change in strain



Fine Aggregate Coarse Aggregate

Figure 12. Typical fracture patterns of wearing course samples.

rate, while for others the strain rate increased rapidly before failure (Fig. 6). This indicates that, although the asphalt was from the same supplier and of the same penetration grade, differences in rheologic properties exist that may affect susceptibility to transverse cracking and which are not possible to explain in terms of stiffness modulus.

The difference in strain patterns could be due to non-homogeneity, differences in strength between aggregate and binder, uneven stress distribution or stress concentrations, or the glass transition temperature of the samples could have varied about the test temperature. In any event the significance of this difference is that regardless of the strain pattern before failure, the only stress-strain relationship of significance to transverse cracking is at break. Therefore, stiffness modulus can be calculated as total stress over total strain at break as a function of time and temperature as suggested by Hajek and Haas in their discussion of Ref. (6). Since total strain at break is more difficult to accurately determine for materials whose strain rate increases before failure, total strain at failure should be defined as the point of deviation from linear viscous creep as suggested in Ref. (7).

It has been rationalized that thermal cracking develops when thermally induced stress σ_{th}, is equal to or just exceeds breaking strength, σ_{br}. To determine thermally induced stress the following equation for elastic materials is used:

$$\sigma_{th} = \alpha E (t - t_0) \qquad (6)$$

α = coefficient of expansion

E = modulus

t = final temperature

t_0 = initial temperature

For thermal cracking, the modulus may be determined by the creep test in which stress is a constant and the strain is that which occurs at the time of break. Then the modulus determined from creep test results can validly be substituted in Eq. (6), only when $\alpha(t - t_0)$ is equal to the breaking strain in which case $\sigma_{th} = \sigma_{br}$. The use of Eq. (6) at strains other than breaking strain should not result in valid calculations of σ_{th} if the modulus used is determined from creep tests. Rather than working with stress and modulus values it would appear simpler to work only with strains. The strain produced by a given drop in temperature over a given time interval can be calculated on the basis of the coefficient of expansion-contraction (α) i.e., thermally induced strain $-\alpha(t - t_0)$. If this strain is equal to or exceeds

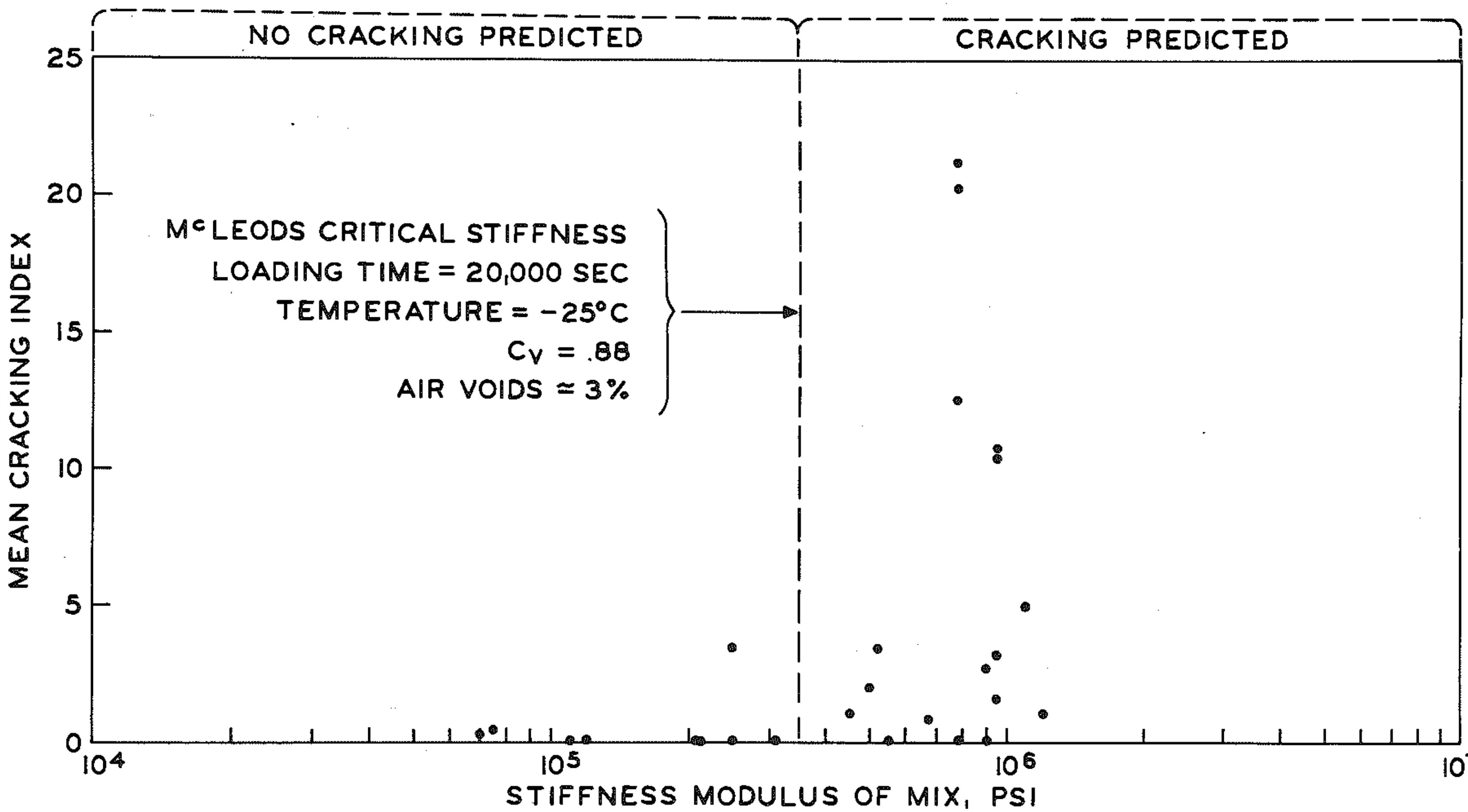


Figure 13. Pavement stiffness modulus vs. mean cracking index, indicating the significance of McLeod's critical stiffness limit.

the strain at break, as determined by the creep test method for the same time interval $(t - t_0)$, then the pavement should crack. It is suggested then that the determination of the temperature below which cracking should occur be based on strain values instead of stress which is the more usual approach.

As far as is known, investigators have not been able to develop a relationship between stiffness modulus of field aged bituminous mixtures and transverse cracking. Some reasons for this are as follows:

1) There is no direct correlation between stiffness modulus and transverse cracking.

2) The stiffness modulus values reported are inaccurate. At low temperatures bituminous mixture breaking strains are so small that they can accurately be determined only with extremely sensitive LVDT instruments. Based on results of our testing it is recommended that strains be measured directly with instruments accurate and readable to 10^{-6} in.

3) The temperature at which the samples are tested and the selection of the breaking point time for determining stiffness modulus are arbitrary and may not reflect field conditions at which transverse cracking takes place.

4) Bituminous concrete pavements are usually constructed in multilayers, the rheologic properties of which vary for each layer. The use of a single stiffness modulus to describe the multilayer system is subject to considerable inaccuracy.

Even though investigators have not been able to establish a direct relationship between the stiffness modulus of an asphalt mix, as determined by direct test and transverse cracking frequency, many investigators feel that the stiffness properties of an asphalt do control its susceptibility to thermal cracking. This feeling is probably due to the fact that replacement of stiff penetration grades of asphalt by those of softer grade will essentially eliminate transverse cracking. For the Michigan pavements evaluated in this study, the relationship between cracking index and estimated asphalt cement stiffness indicates that transverse cracking could have been practically eliminated had the maximum stiffness limit suggested by McLeod (5) been used to select asphalt penetration grades (Fig. 13). It was noted also, however, that many pavements constructed using asphalt of a stiffness greater than McLeod's suggested limit are not subject to cracking either. It appears, therefore, that transverse cracking is not a function of stiffness

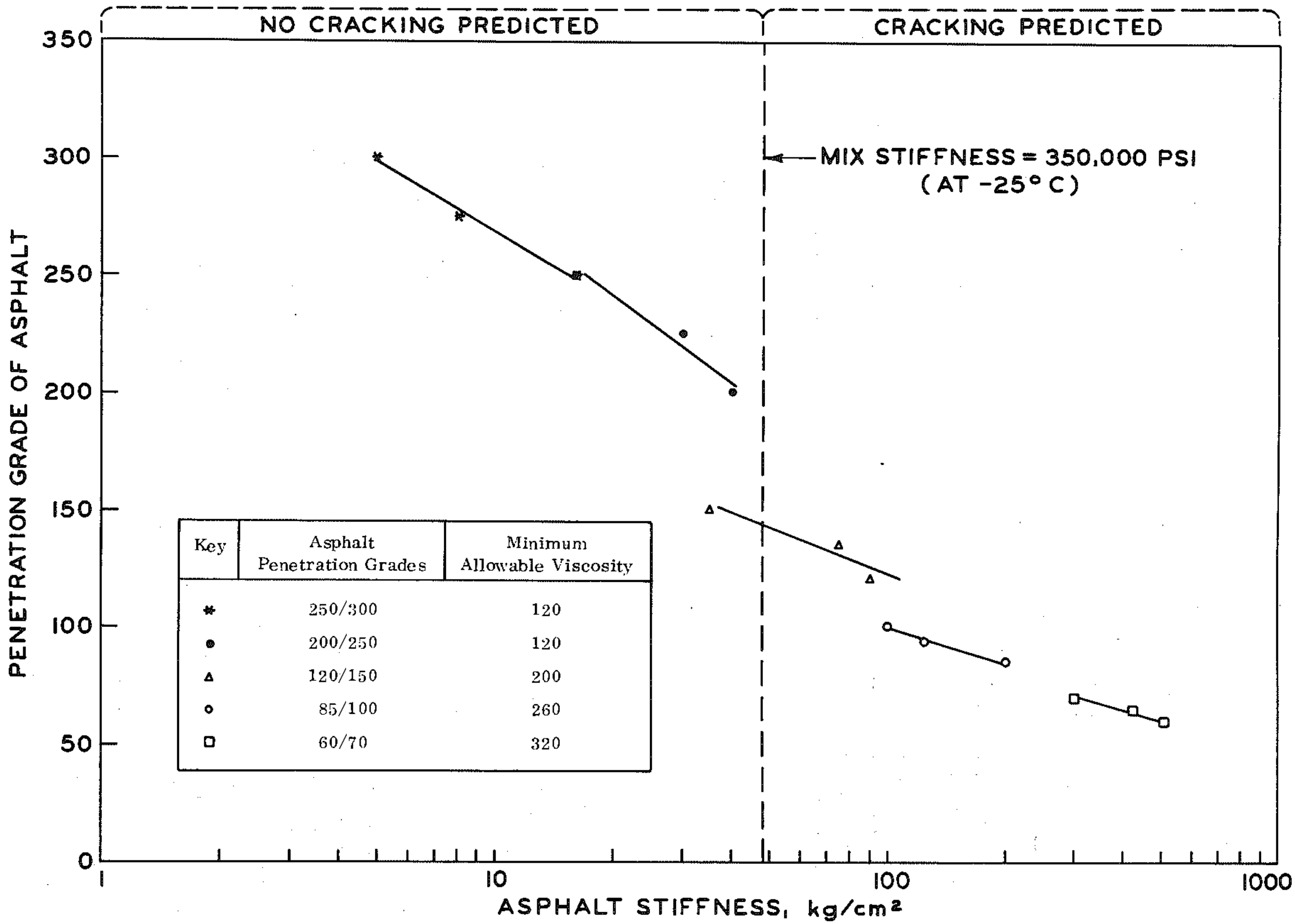


Figure 14. Michigan's standard asphalt penetration grades at minimum allowable viscosity, vs. their asphalt cement stiffness modulus at -20 C and 20,000 sec loading time.

modulus alone. The use of increasingly soft asphalt apparently diminishes the influence of other asphalt properties to a point where these properties have little or no influence. That stiffer asphalts can also perform well, is clearly indicated by Figure 13 and an examination of the area listed as observation 34 (Table 1) which was constructed using 60 to 70 penetration grade asphalt and is essentially crack-free after 12 years of service. When applied to Michigan's specification asphalt cements, McLeod's criteria showed that only penetration grades of 200 or above could be expected to be essentially crack-free, but for grades below 200, cracking may be expected (Fig. 14).

Transverse cracking is only one form of flexible pavement cracking, all forms of which are dependent on rheologic and fracture properties of the bituminous mixes as placed and as affected by field use and aging. For this reason a long-term rational approach to the entire interrelated asphalt cracking problem might be an eventual best solution to the problems. A factor complicating rational study of flexible pavement cracking characteristics is their simultaneous exposure to thermal and traffic induced stresses, the combination of which could significantly affect transverse cracking potential. Although rational studies should be most fruitful, empirical or statistical approaches to specific problems could be useful for expediency where applicable.

Some investigators have questioned the value of considering cracking frequency at all because pavements can be designed to be crack-free (6). In studying cracking on this project it became apparent that a mean cracking index for a given area can be misleading. Table 1 shows that the mean cracking index is often smaller than its standard deviation. Where cracking index values are to be used it is suggested that their standard deviation be also reported so that some idea of their uniformity can be realized.

SUMMARY AND CONCLUSIONS

In this study the Hajek-Haas model, designed to predict transverse cracking susceptibility of flexible pavements, was statistically evaluated by testing its ability to predict the mean cracking index of selected Michigan pavements. A supplemental study was conducted when it was found that the model was not suitable, in an effort to determine why the model failed and to provide direction for possible future work in this area. These studies have produced the following observations and conclusions.

1) For Michigan's flexible pavements, the Hajek-Haas model lacks the ability to predict, within reason, transverse cracking performance.

2) Modification of the model, based on data collected in Michigan, failed to improve predictive ability sufficiently to warrant its use.

3) Of the model's five independent variables, only stiffness modulus was significantly related to the cracking frequency of Michigan flexible pavements.

4) Rational studies of transverse cracking indicate that it is the bituminous concrete stiffness modulus, tensile strength, and coefficient of expansion properties acting in combination with climatic conditions, that governs an asphalt's susceptibility to transverse or thermal cracking.

5) The Hajek-Haas model demonstrated poor predictive ability because it included only one of several bituminous concrete properties which affect transverse cracking.

6) No direct correlation could be found between cracking index and the following stiffness modulus characteristics:

a. The modulus of field aged bituminous concrete determined by creep test;

b. The estimated modulus of field aged asphalt cement determined using McLeod's method of estimating modulus from penetration viscosity data;

c. The estimated modulus of field-aged bituminous concrete determined using Heukelom and Klomps chart for converting estimated cement stiffness to mix stiffness on the basis of the volume concentration of the aggregate, C_V.

7) The stiffness modulus of asphalt cement can be lowered to such an extent that other properties of the bituminous mix which also influence transverse cracking are diminished to a non-effective level.

8) The ability of very soft asphalts to override other mix properties apparently enable criteria to be established that will permit designers to essentially eliminate transverse cracking based on the estimated stiffness of the asphalt cement.

9) Based on the transverse cracking performance of Michigan's flexible pavements, McLeod's method of selecting penetration grades of asphalt that will produce essentially crack-free pavements is conditionally recommended depending on the cost and availability of soft asphalt grades and the ability to develop mix designs of suitable high temperature stability.

10) The data collected for this study indicate that stiff grades of asphalt, 60 to 70 penetration grade, can also perform in a crack-free manner. Because of this and the fact that harder asphalt cements are desirable from a stability standpoint, there is a need to develop a method of more accurately assessing the susceptibility of bituminous mixes to transverse or thermal cracking.

11) Michigan flexible pavements consist of three separately constructed layers of bituminous concrete. For convenience the total bituminous concrete layer is usually considered homogeneous and isotropic for most design purposes. However, this study indicates this assumption to be incorrect since the rheologic and fracture properties of each layer differ significantly. This increases the difficulty with which the properties of the bituminous layer can be characterized and, of course, increases the difficulty of predicting all types of surface cracking.

12) The tensile strength of bituminous mixes, at low temperature, is dependent on the tensile strength of the aggregate as indicated by the large percentage of tensile failure through the aggregate itself.

In Michigan, surface cracking is of primary concern because it is the predominant form of flexible pavement failure. High temperature design procedures are well developed and effective, but design procedures to control cracking are almost non-existent. The need exists for development of methods of designing flexible pavements that are essentially crack-free. In this respect, all forms of flexible pavement cracking are interrelated and, therefore, transverse cracking should be treated as a special form of the overall cracking problem.

Thermal and load induced tensile stresses act in combination so that improving a pavement's resistance to thermal cracking should also improve its resistance to cracking caused by load-induced stresses. In addition, load-induced tensile stress may act in combination with thermal stress to cause transverse and other forms of cracking.

On the basis of this study the rational approach to the thermal cracking problem suggested by Burgess, et al, appears to be the most promising at the present time (6). In this respect it is more conventional to deal with such a problem on the basis of stress. However, this study indicates that it may be easier and more realistic to use strains to estimate the temperature at which thermal cracking may develop.

A committee within the Department has been appointed to review the results of this project and recommend possible methods for implementing any findings which may have application to asphalt surfacing problems.

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